TRANSMITTAL OF SHOP DRAWINGS, EQUIPMENT DATA, MATERIAL SAMPLES, OR MANUFACTURER'S CERTIFICATES OF COMPLIANCE
For use of this form, see ER 415-1-10; the proponent agency is CECW-CE.

DATE: 06 / 15 / 2021
DELIVERY VIA Email

TRANSMITTAL NO. **TA – 031**

SECTION I - REQUEST FOR APPROVAL OF THE FOLLOWING ITEMS (This section will be initiated by the contractor)

TO:	FROM:	CONTRACT NO.	CHECK ONE:
NAVFAC	JOHN C. GRIMBERG CO., INC. 3200 Tower Oaks Blvd., Suite 300 Rockville, MD 20852	N4008018C0006 JOHN C. GRIMBERG, CO., INC.	☒THIS IS A NEW TRANSMITTAL ☐THIS IS A RESUBMITTAL OF TRANSMITTAL ____

SPECIFICATION SEC. NO. (Cover only one section with each transmittal)	PROJECT TITLE AND LOCATION	THIS TRANSMITTAL IS FOR: (Check one)
CAP 030, No MDL # – Mis-Located ICF reinforcement bars.	P021 Middle School/High School Replacement, Marine Corps Base Quantico, VA (#879)	☐FIO ☒GA ☐DA ☐CR ☐DA/CR ☐DA/GA

ITEM NO. (See Note 3) a.	DESCRIPTION OF SUBMITTAL ITEM (Type, size, model number/etc.) b.	SUBMITTAL TYPE CODE (See Note 8) c.	NO. OF COPIES d.	CONTRACT DOCUMENT REFERENCE SPEC. PARA. NO. e.	CONTRACT DOCUMENT REFERENCE DRAWING SHEET NO. f.	CONTRACTOR REVIEW CODE g.	VARIATION Enter "Y" if requesting a variation (See Note 6) h.	FOR ACTION CODE (Note 9) i.
1	**WDP cover sheet and TT calculations PE stamped and sealed, PDF pages 2- 16**	**SD-05 Design data**	**PDF Electronic**			**A**	**YES**	X

Action Code "X" - Receipt acknowledged, does not comply with contract requirements

Remarks: During the CAP 13 ICF Wall void investigation, reinforcement was exposed that was not in the correct plane within the typical ICF wall thickness. To address the possibility that this condition exists elsewhere, the analysis in this submittal assumes discrepancies (i.e., assumes vertical wall reinforcement has drifted to mid-depth at mid-height) and justifies that, if such conditions existed, walls with such conditions would have sufficient load carrying capacity to resist the loads prescribed in the contract documents and the applicable design loads provided by EwingCole. The submittal includes applicable design calculations (signed and sealed) to support this analysis.

I certify that the above submitted items have been reviewed in detail and are correct and in strict compliance with the contract drawings and specifications except as otherwise stated.

Stephen Boyles
John C. Grimberg Co., Inc.
NAME OF CONTRACTOR

Stephen Boyles
SIGNATURE OF QCM

SECTION II - APPROVAL ACTION

ENCLOSURES RETURNED (List by Item No.)	NAME AND TITLE OF APPROVING AUTHORITY	SIGNATURE OF APPROVING AUTHORITY	DATE
	LT BEERING, CM	BEERING.SAMUEL.CONYERS.1500472789 Digitally signed by BEERING.SAMUEL.CONYERS.1500472789 Date: 2021.07.01 13:24:17 -04'00'	1JUL2021

ENG FORM 4025-R, MAR 2012 REPLACES EDITION OF MAR. 95, WHICH IS OBSOLETE.

1. Wall slenderness has not been considered.

2. Combined effects of axial loads with moment have not been considered.

3. As-built wall out of plumbness has not been considered.

4. Table on sheet 5 tabulates "unbraced wall height" for several conditions. These heights often appear to assume that the slab on grade braces the wall, which is not the case:
   A.) Bracing points should be considered at the following locations:
      i.) Top of footing (walls are not positively anchored to the slab on grade)
      ii.) 2nd floor slab on deck, where applicable
      iii.) Metal roof deck
   B.) At two-span conditions it does not appear that the longer span is always considered.

5. Per table on sheet 5, the following cases appear to be considered:
   A.) Generic 8" wall with 20' span and no openings
   B.) Generic 12" wall with 40' span and no openings
   C.) Specific walls and piers (if applicable) along Grids P, 6, M, J.5, A & K
   Additional specific checks are necessary along the following grids: U, V, U.2, C.1, 17, 7, 6.1, L, and longspan 8" walls at grids B/7 and B/17.

6. Generic 8" and 12" cases appear to be intended to cover walls without pairs of openings that create a narrow pier. Moment caused by eccentricity of gravity loading does not appear to be considered for these cases.

7. Table on sheet 6 notes each grid location as a typical wall or corner. Grids M & A should be analyzed with corner wind loads.

8. Sheets 7 & 8 address moment due to gravity load eccentricity at joist bearing seats, however gravity load eccentricity at beam bearing pockets and embed plate connections do not appear to be addressed. Additionally, only roof load is considered, but the span between foundation and 2nd floor is often the controlling span for wind moment design, and therefore second floor loading should also be considered.

9. Sheet 8 notes load combination of 1.2D+0.2S+W. Per ASCE 7-10 the appropriate load factor for snow is 0.5.

10. Sheet 10 Table column for "Max Moment from Joist Ecc - Wind": it is unclear how these values are derived; they do not appear to correspond to the moments calculated on Page 8.

11. Sheet 10 states "The reinforcement spacing and cover at locations above with a DCR greater than 1.0 were verified in the field. The actual cover was less than the worst-case scenarios previously analyzed, and the as-built reinforcement at the critical sections was adequate such that there are no issues at those locations."
   A.) It is unclear what method was used to determine as-built cover
   B.) As-built cover survey data was not provided
   C.) Calculations were not provided to support the conclusion that as-built cover is adequate.

12. Analysis for the wall on Grid 28 states that the wall is adequate with a maximum clear cover of 3", and that the as-built cover satisfies this at all but one location:
   A.) As-built clear cover survey information has not been provided.
   B.) Calculations have not been provided to justify the statement that 3" clear cover is adequate at all locations.

13. Sheet 11: calculated shear capacity does not appear to apply a strength reduction factor.

14. Sheet 11 states that at grid 28 horizontal reinforcement is spaced at 18" (9" is specified on drawings). SW-28 must be evaluated for the increased spacing.

15. Note that ACI 318-11 Chapter 14.3.4 requires for walls with thickness greater than 10", reinforcement must be placed in two layers with one layer no more than 1/3*thickness from the interior face and the other layer no more than 1/3*thickness from the exterior face. The information in this submittal indicates that this provision is violated at Grid 28. Clear cover should be verified at all 12" walls.

The reinforcement placement discrepancies addressed in this submittal are being considered in combination with other reinforcement discrepancies observed in the field. The method for evaluation of all reinforcement as-built conditions is to be determined.

SUBMITTAL ACTION

SUBMISSION ☒ FIRST ☐ SECOND ☐ THIRD

- ☐ REVIEWED
- ☐ REVIEWED AS NOTED
- ☐ REVIEWED AS NOTED RESUBMIT
- ☐ REJECTED
- ☐ RESUBMIT
- ☒ SEE TRANSMITTAL FOR INSTRUCTIONS

This submittal has been reviewed only for conformance with the design concept of the Project and compliance with the information given in the Contract Documents.

It is the Contractor's contractual duty to review and act upon submittals, prior to their submission to verify that all requirements of the Contract Documents have been met or if they have not been met, to notify the Architect in writing. By the submission of this submittal to the Architect, it is assumed the Contractor has fulfilled these contractual duties. Review of this submittal by Architect does not relieve the Contractor of the duty to meet the requirements of the Contract Documents and the applicable building codes.

Any comments noted or corrections requested are for clarification of the general character of the work. Correctness of details, measurements, quantities, conformity with Contract Documents, techniques of construction and coordination with other trades shall remain the complete responsibility of the General/Prime Contractor.

BY: MES DATE: 07/01/21

EWINGCOLE
Federal Reserve Bank Building, 100th N 6th Street
Independence Mall West Philadelphia, PA 19106
ARCHITECTS ENGINEERS INTERIOR DESIGNERS PLANNERS

Corrective Action Plan Submittal
Quantico Middle/High School

Date: June 14th, 2021

To: Steve Boyles, P.E., CCM, CQM
Quality Manager
John C. Grimberg Co., Inc.

From: Eric Peterson, P.E., Principal
WDP & Associates Consulting Engineers, Inc.

Manassas, VA
Charlottesville, VA
Blacksburg, VA
Myrtle Beach, SC
New York, NY

Corrective Action Plan Submittal Number: 31
Corrective Action Plan Description: ICF Reinforcement
Master Discrepancy List (MDL) Number: N/A

Mr. Boyles,

Attached, please find the corrective action plan resubmittal for CAP 31 - ICF Wall Reinforcement for the above referenced project. This submittal was composed by Thorton Tomasetti and has been reviewed by our office, in conjunction with the John C. Grimberg project team for its general concept. Specific comments provided by the SEOR and NAVFAC are discussed below. Please review the information provided under this cover and contact our office in the event of any questions or conflicts. Thank you.

Sincerely,

WDP & Associates Consulting Engineers, Inc.

J. Eric Peterson, P.E.
VA PE #0402028879

COMMONWEALTH OF VIRGINIA
J. ERIC PETERSON
Lic. No. 28879
PROFESSIONAL ENGINEER

CC: B.J. Lee
Andrew Blasetti
Henry Danforth
Steve Vanderwoude
Calvin Austin
John Wells
Justice O'Connor

10621 Gateway Blvd., Suite 200, Manassas, Virginia 20110 | tel: 703.257.9280 | fax: 571.292.9842 | www.wdpa.com

Thornton Tomasetti

PROJECT: Quantico MS/HS		PROJECT #: L20068	DATE: 06/01/21
SUBJECT:		BY: VAB	SHEET: 1 of
ICF Wall Summary - Calcs		CHECKED BY: AB	DRAWING #:

The ICF walls have conditions that indicate potential shifting of reinforcement within the walls. The walls were analyzed for loads provided by the DoR. The following calculations for the ICF walls include:

1. Capacity of ICF walls, considering misplaced wall reinforcement.
2. Moments induced from transverse loads.
3. Moments induced from joist eccentricity (vertical reactions).

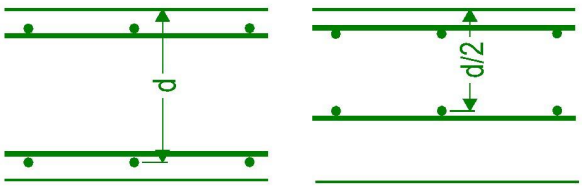
Thornton Tomasetti

PROJECT: Quantico MS/HS		PROJECT #: L20068	DATE: 06/01/21
SUBJECT:		BY: VAB	SHEET: 2 of
ICF Wall Summary - Calcs		CHECKED BY:	DRAWING #:

The flexural capacity of the walls was calculated based on the reinforcement detailed in the '"Revised" Insulating Concrete Form - shop drawings (ICF Walls)' document dated 07/20/2018. The dimensions and reinforcing for each wall are shown in the table below.

Level	Area	Grid	Unbraced Wall Height (ft)	Wall Thickness (in)	Vertical Rebar	Horizontal Rebar	Clear Cover (in)
Low Roof	Any	Any	20.00	8	#4@8" EF	#5@18" EF	1.5
Main Roof	Any	Any	40.00	12	#5@8" EF	#5@18" EF	1.5
Second Floor &	A	P	16.83	8	#4@8" EF	#5@18" EF	1.5
		6	14.42	8	#4@8" EF	#5@18" EF	1.5
Main Roof	A	M	35.00	12	#5@8" EF	#5@18" EF	1.5
	B	J.5	12.40	8	#4@8" EF	#5@18" EF	1.5
	D&E	A	12.63	8	#4@8" EF	#5@18" EF	1.5
Gable Roof	A&B	K	12.63	8	#4@8" EF	#5@18" EF	1.5

Due to the potential that wall reinforcement shifted while being installed, the bending capacity was checked for both the full design depth of the wall, as well as for a reduced depth. The full depth considers the reinforcement properly placed per the shop drawings. The reduced depth assumes the reinforcement has drifted to mid-depth at mid-height, and the compression side bars do not contribute. See below sketch representing the two analyses.



The value for "d" was calculated per the following:

d = Wall Thickness (in) - 1.5" Cover - 1/2(Vertical Bar Diameter)

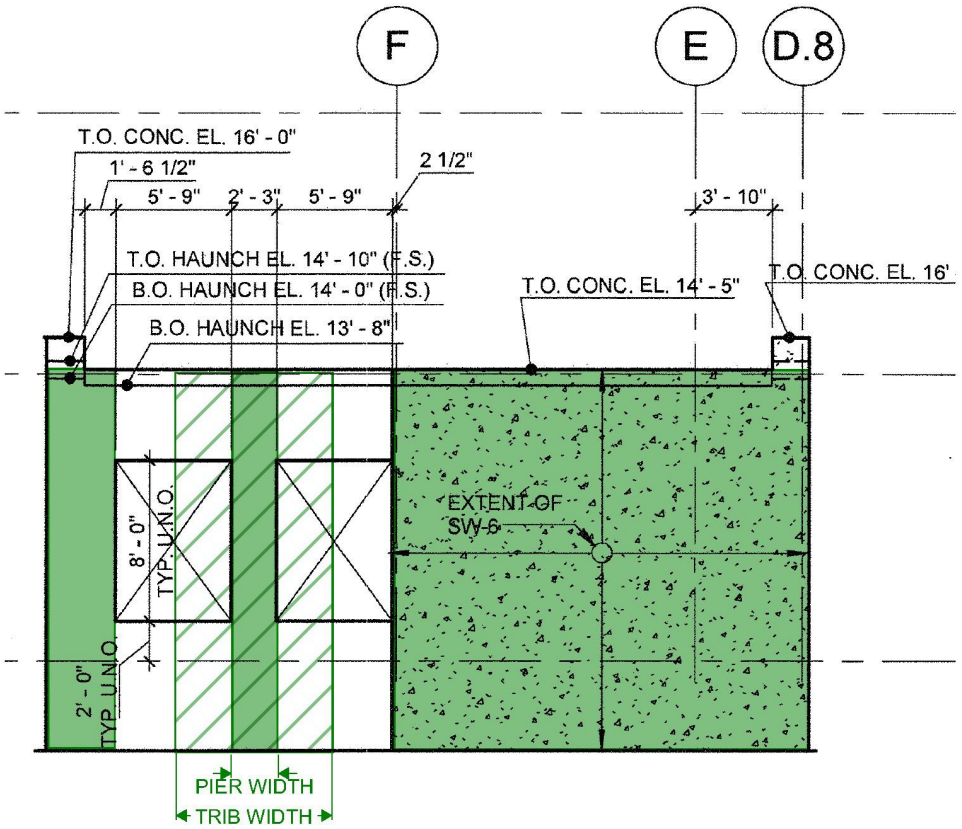
The results are shown below for the 8" and 12" thick walls.

Wall Thickness	Vertical Bar Size	Distance from Edge of Wall to Rebar (in)			
		d		d/2	
8"	#4 (0.5" dia)	8" - 1.5" (cover) - 0.5*0.5" =	6.25	1/2*6.25 =	3.13
12"	#5 (0.625" dia)	12" - 1.5" (cover) - 0.5*0.625" =	10.19	1/2*10.19 =	5.09

Thornton Tomasetti

PROJECT: Quantico MS/HS		PROJECT #: L20068	DATE: 06/01/21
SUBJECT:		BY: VAB	SHEET: 3 of
ICF Wall Summary - Calcs		CHECKED BY:	DRAWING #:

In addition to analyzing the walls on a per-foot basis, most walls have multiple windows separated by piers. The piers were analyzed for loads acting on the tributary width of wall that is braced by the pier. The smallest pier was chosen for analysis along each wall, as that would be the worst case loading. See figure below for reference of how the "Width (ft)" and "Tributary Width (ft)" were calculated for the following capacity checks.

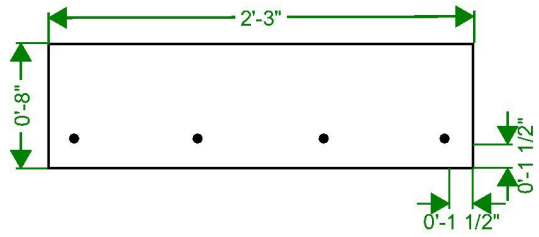


6 ICF WALL ELEVATION ALONG GRID 6

Thornton Tomasetti

PROJECT: Quantico MS/HS		PROJECT #: L20068	DATE: 06/01/21
SUBJECT:		BY: VAB	SHEET: 4 of
ICF Wall Summary - Calcs		CHECKED BY:	DRAWING #:

Using the wall along Grid 6 as an example, the calculations for the shear and moment capacity for the walls are shown below.



f'c = 4000psi
fy = 60ksi
Pier Width = b = 2'-3" (27")
Wall Thickness = 8"
Concrete Cover = 1.5"
Vertical Reinforcement = #4 bars @ 8" c/c
#4 Bar Diameter = 0.5"
#4 Bar Area = 0.2in^2

d = 8" - 1.5" - 1/2*0.5" = 6.25"

Shear Capacity

Φv = 0.75

ΦvVc = Φv * 2 * min(100, sqrt(f'c)) * b * d / 1000
ΦvVc = 0.75 * 2 * min (100, sqrt(4000psi)) * 27" * 6.25" / 1000
ΦvVc = 16.0k

Moment Capacity

Φb = 0.90
As,total = 4 bars * 0.2in^2 = 0.80in^2

a = As,total * fy / (0.85 * f'c * b)
a = 0.80in^2 * 60ksi / (0.85 * 4ksi * 27")
a = 0.523"

ΦbMn = Φb * As,total * fy * (d - a/2) / 12
ΦbMn = 0.90 * 0.80in2 * 60ksi * (6.25" - 0.523"/2) / 12"
ΦbMn = 21.6k-ft

For d/2:

ΦvVc = 0.75 * 2 * min (100, sqrt(4000psi)) * 27" * 6.25/2" / 1000 = 8.0k

ΦbMn = 0.90 * 0.80in2 * 60ksi * (6.25"/2 - 0.523"/2) / 12" = 10.3k-ft

Thornton Tomasetti

PROJECT: Quantico MS/HS | PROJECT #: L20068 | DATE: 06/01/21

SUBJECT: | BY: VAB | SHEET: 5 of

ICF Wall Summary - Calcs | CHECKED BY: | DRAWING #:

Each wall was analyzed using the same equations on the previous page and the results are shown in the table below. Additional vertical #7 bars at openings per the typical details were conservatively neglected in the initial calculations, and only considered where verified in the field.

Grid	Unbraced Wall Height (ft)	Wall Thickness (in)	Pier						Wall			
					d		d/2		d		d/2	
			Width (ft)	Trib Width (ft)	Shear Capacity (k)	Moment Capacity (k-ft)	Shear Capacity (k)	Moment Capacity (k-ft)	Shear Capacity (k/ft)	Moment Capacity (k-ft/ft)	Shear Capacity (k/ft)	Moment Capacity (k-ft/ft)
Any	20.00	8	N/A						7.1	8.1	3.6	4.1
Any	40.00	12	N/A						11.6	20.6	5.8	9.9
P	16.83	8	2.25	8.67	16	21.6	8	10.3	-			
6	14.42	8	2.25	8	16	21.6	8	10.3	-			
M	35.00	12	N/A						11.6	20.6	5.8	9.9
J.5	12.40	8	7.25	13.67	51.6	59	25.8	28.4	-			
A	12.63	8	2.25	8	16	21.6	8	10.3	-			
K	12.63	8	2.25	8.67	16	21.6	8	10.3	-			

Thornton Tomasetti

PROJECT: Quantico MS/HS		PROJECT #: L20068	DATE: 06/01/21
SUBJECT:		BY: VAB	SHEET: 6 of
ICF Wall Summary - Calcs		CHECKED BY:	DRAWING #:

Bending moments due to wind loads were obtained from the notes on S003 in the Structural Drawings, with the table shown below.

WIND LOADS ON COMPONENTS AND CLADDING:

COMPONENT LOCATION **	
TYPICAL WALL	33 PSF
WALL CORNERS	40 PSF
TYPICAL ROOF	38 PSF
ROOF PERIMETER	48 PSF
ROOF CORNER	78 PSF

** NOTE: COMPONENT WIND LOADS ARE BASED ON A TRIBUTARY AREA OF 10 SQ. FT. VALUES MAY BE ADJUSTED PROVIDED WIND LOAD CALCULATIONS ARE SUBMITTED FOR REVIEW.

WIND LOADS ON JOISTS: SEE PLAN 1 ON THIS SHEET FOR NET WIND UPLIFT ON JOISTS.

The unbraced wall height of each wall was determined based on the structural drawings to determine the internal forces due to wind loading.

Moment Demand = Wind Load per Foot (plf) * Unbraced Wall Height (ft) ^2 / 8

The resulting bending moment applied to the ICF walls is shown in the table below.

Area	Grid	Wind Load on Wall			
		Location	Wind Load per Foot	Moment (k-ft/ft)	Shear (k/ft)
Any	Any	C	40	2.00	0.33
Any	Any	C	40	8.00	0.8
A	P	T	33	1.17	0.62
	6	C	40	1.04	0.65
A	M	T	33	5.05	0.58
B	J.5	T	33	0.63	1.48
D&E	A	T	33	0.66	0.47
A&B	K	T	33	0.66	0.47

The seismic loading for the building was provided on S003 in the Structural Drawings. The value of Cs=0.0578 was multiplied by the weight of the wall: 8" (100 psf) or 12" (150 psf) thickness, plus assumed 50 psf cladding.

Weq = 200 psf * 0.0578 = 11.6 psf < 33 psf min wind load

Thornton Tomasetti

PROJECT: Quantico MS/HS		PROJECT #: L20068	DATE: 06/01/21
SUBJECT:		BY: VAB	SHEET: 7 of
ICF Wall Summary - Calcs		CHECKED BY:	DRAWING #:

The eccentricities of the joists on the ICF wall were calculated and reported in CAP 07.1, CAP 07.2, and CAP 07.3. The eccentricity of each joist with respect to the centerline of the wall was determined, with the worst-case eccentricity for each ICF wall is shown below.

Level	Area	Grid	Max Joist Eccentricity (in)	Reference
Second Floor & Low Roof	A	P	2	CAP 07.2
		6	2	CAP 07.2
Main Roof	A	M	2	CAP 07.2
	B	J.5	4.3125	CAP 07.3
	D&E	A	2.5	CAP 07.2
Gable Roof	A&B	K	4.75	CAP 07.1

Bending moment applied to the wall due to this eccentricity was determined via the loads provided in the QMHS_concrete connection loads_03-15-21 document for Figure 16 - Concrete Connection Type J. Figure 16 is shown on p.9, and the loads applied to each wall are shown in tabular format below.

Level	Area	Grid	Figure 16 Gravity Loads			
			D (k)		Snow (k)	Factored Wind (k)
			Max	Min		
Second Floor & Low Roof	A	P	3	0.5	5	8
		6	3	0.5	5	8
Main Roof	A	M	3	0.5	5	8
	B	J.5	3	0.5	5	8
	D&E	A	3	0.5	5	8
Gable Roof	A&B	K	3	0.5	5	8

Thornton Tomasetti

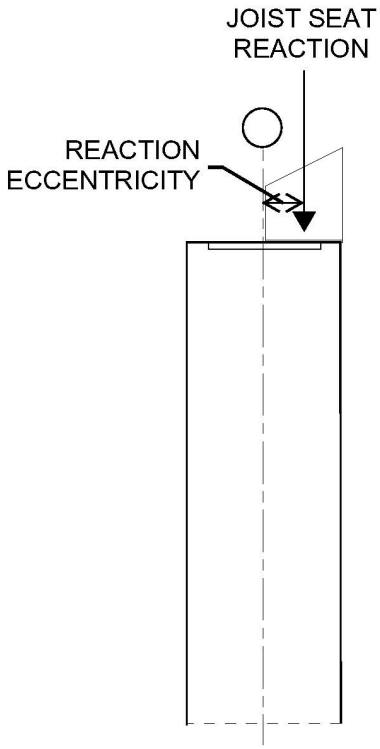
PROJECT: Quantico MS/HS		PROJECT #: L20068	DATE: 06/01/21
SUBJECT:		BY: VAB	SHEET: 8 of
ICF Wall Summary - Calcs		CHECKED BY:	DRAWING #:

The factored loads were calculated as follows:

Moment due to Wind (down) = 1.2*Dead + 0.2*Snow + Wind
Moment due to Wind (up) = 0.9*Dead + Wind

The resulting load was then multiplied by the joist eccentricity to calculate the applied bending moment, with the results shown in the table below. The net uplift plus flexure condition does not control the design.

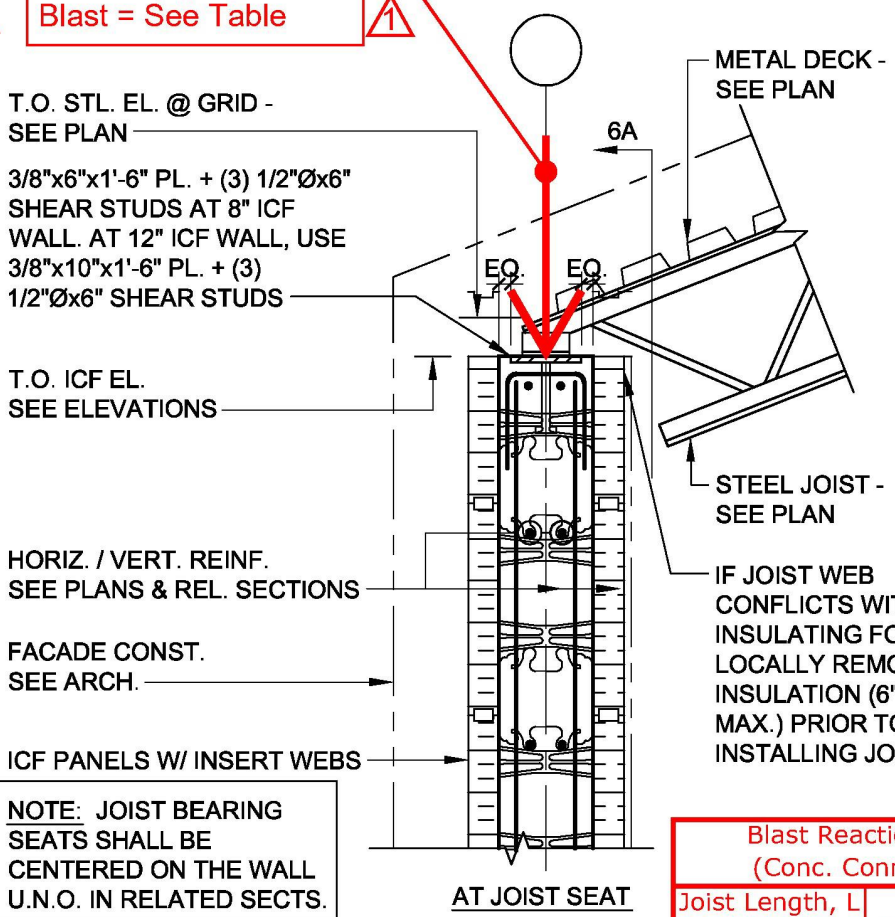
Area	Grid	Worst-Case Eccentricity from Joists				
		Joist Eccentricity	Wind-down (k)	Moment (k-ft)	Wind-up (k)	Moment (k-ft)
Any	Any	-	-	-	-	-
Any	Any	-	-	-	-	-
A	P	2	12.6	2.10	7.55	1.26
	6	2	12.6	2.10	7.55	1.26
A	M	2	12.6	2.10	7.55	1.26
B	J.5	4.3125	12.6	4.53	7.55	2.71
D&E	A	2.5	12.6	2.63	7.55	1.57
A&B	K	4.75	12.6	4.99	7.55	2.99



Note from TT:
Page from QMHS_concrete connection loads_03-15-21
Provided by Ewing Cole
For reference only

D = 0.5 k min, 3 k max
S = 5 k
W = 8 k (uplift)
Blast = See Table

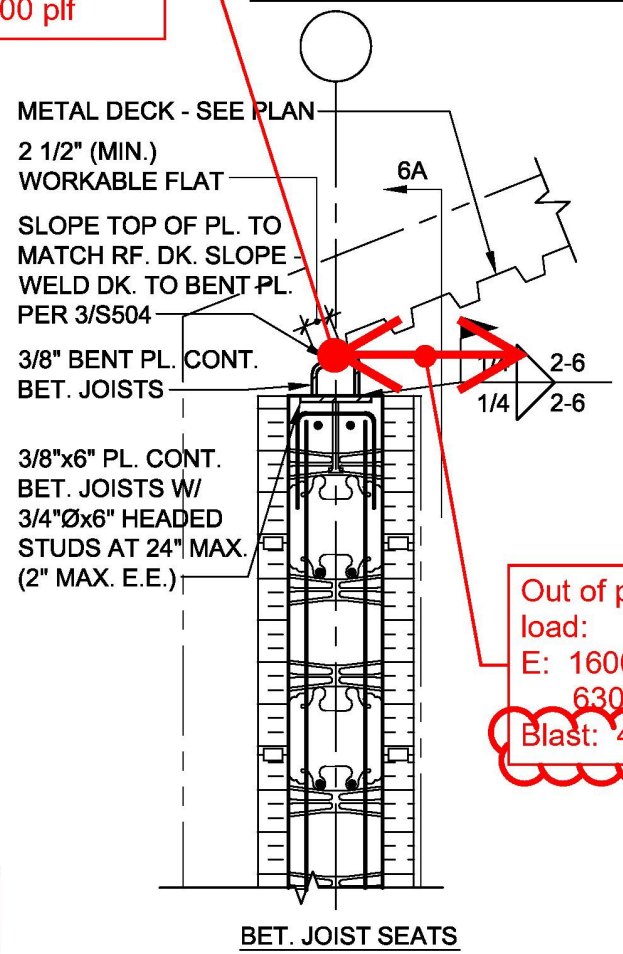
(into/out of page):
W = 1100 plf
E = 1200 plf



AT JOIST SEAT

NOTE: JOIST BEARING SEATS SHALL BE CENTERED ON THE WALL U.N.O. IN RELATED SECTS.

Blast Reaction at Joist (Conc. Conn. Type J)	
Joist Length, L (ft)	Blast Rxn (k, up or down)
L ≤ 20	8.5
20 < L ≤ 30	12
L > 30	17.4



BET. JOIST SEATS

Out of plane wall bracing load:
E: 1600 plf at 12" wall
630 plf at 8" wall
Blast: 400 plf (typ.)

Note: Lateral drag load and out of plane wall bracing load do not occur simultaneously.

Section at Joist Seat & Deck Support on Top of Wall

(Reference Typical Detail 6/S503)

Note:
Gravity loads indicated on this figure are unfactored, service level loads. Wind and Seismic loads are strength level loads as generated by ASCE 7-10. Verification of as-built capacities must consider all applicable ASCE 7-10 LRFD load combinations.

FIGURE 16

Concrete Conn. Type J

Thornton Tomasetti

PROJECT: Quantico MS/HS | PROJECT #: L20068 | DATE: 06/01/21

SUBJECT: | BY: VAB | SHEET: 10 of

ICF Wall Summary - Calcs | CHECKED BY: | DRAWING #:

See results below for the wall capacities and DCR ratios.

Grid	Unbraced Wall Height (ft)	Wall Thickness (in)	Pier						Wall			
					d		d/2		d		d/2	
			Width (ft)	Trib Width (ft)	Shear Capacity (k)	Moment Capacity (k-ft)	Shear Capacity (k)	Moment Capacity (k-ft)	Shear Capacity (k/ft)	Moment Capacity (k-ft/ft)	Shear Capacity (k/ft)	Moment Capacity (k-ft/ft)
Any	20.00	8	N/A						7.1	8.1	3.6	4.1
Any	40.00	12	N/A						11.6	20.6	5.8	9.9
P	16.83	8	2.25	8.67	16	21.6	8	10.3	-			
6	14.42	8	2.25	8	16	21.6	8	10.3	-			
M	35.00	12	N/A						11.6	20.6	5.8	9.9
J.5	12.40	8	7.25	13.67	51.6	59	25.8	28.4	-			
A	12.63	8	2.25	8	16	21.6	8	10.3	-			
K	12.63	8	2.25	8.67	16	21.6	8	10.3	-			

Grid	Unbraced Wall Height (ft)	Wall Thickness (in)	Max Moment from Joist Ecc - Wind (k-ft)	Moment due to Wind on Wall (k-ft)	Total Moment DCR		Total Shear DCR	
					d	d/2	d	d/2
Any	20.00	8	-	2.00	0.25	0.49	0.05	0.09
Any	40.00	12	-	8.00	0.39	0.81	0.07	0.14
P	16.83	8	3.23	10.13	0.62	1.30	0.34	0.68
6	14.42	8	2.94	8.31	0.52	1.09	0.32	0.65
M	35.00	12	2.10	5.05	0.35	0.72	0.05	0.10
J.5	12.40	8	8.20	8.66	0.29	0.59	0.39	0.79
A	12.63	8	3.60	5.26	0.41	0.86	0.23	0.47
K	12.63	8	7.98	5.70	0.63	1.33	0.25	0.51

The capacity of the wall using both the full and reduced depths were checked to ensure that if the reinforcement shifted within the wall during placement, the wall was still able to resist the applied bending. The table shows that all of the ICF walls are capable of resisting the applied moments and shears as designed; however, misaligned reinforcement may cause issues at certain locations with additional moment from the joist eccentricities.

The reinforcement spacing and cover at locations above with a DCR greater then 1.0 were verified in the field. The actual cover was less than the worst-case scenarios previously analyzed, and the as-built reinforcement at the critical sections was adequate such that there are no issues at those locations.

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It is assumed that the wall at Grid 28 relies on two-way action from the continuous vertical and horizontal reinforcement to span wind load around the large openings to the perpendicular walls and diaphragms. Per finite element analysis, the reinforcement and cover indicated on the shop drawings is adequate for the applied moments and shears, for a conservative wind load of 40 psf across the entire elevation, as shown on the next pages.

Per field data, there is a discrepancy with spacing of the horizontal reinforcement, where the as-built spacing is typically 18" at this wall. For this condition, the maximum allowable cover to the reinforcement may be 3" in order for the wall to adequate. This maximum cover was verified in the field at both wall faces with the exception of one location on the interior face, but the as-built quantity and spacing of the vertical reinforcement there is sufficient for the shear and moment per the calculations below.

f'c = 4000psi fy = 60ksi
Wall Thickness = 12" Concrete Cover = 1.5"
Vertical Reinforcement = #5 bars @ 8" c/c
Clear Cover to Vert Reinf = 6" d = 12-6-0.5*0.625 = 5.69"

Shear in Vertical Direction

Vu = 0.8 k/ft (from FEA)

ΦvVc = 0.75 * 2 * min (100, sqrt(4000psi)) * 12" * 5.69" / 1000
ΦvVc = 8.6 k/ft

Moment in Vertical Direction

Mu = 9.5 k-ft/ft (from FEA)

ΦbMn = 0.90 * 0.47in2 * 60ksi * (5.69" - 0.684"/2) / 12"
ΦbMn = 11.2 k-ft/ft

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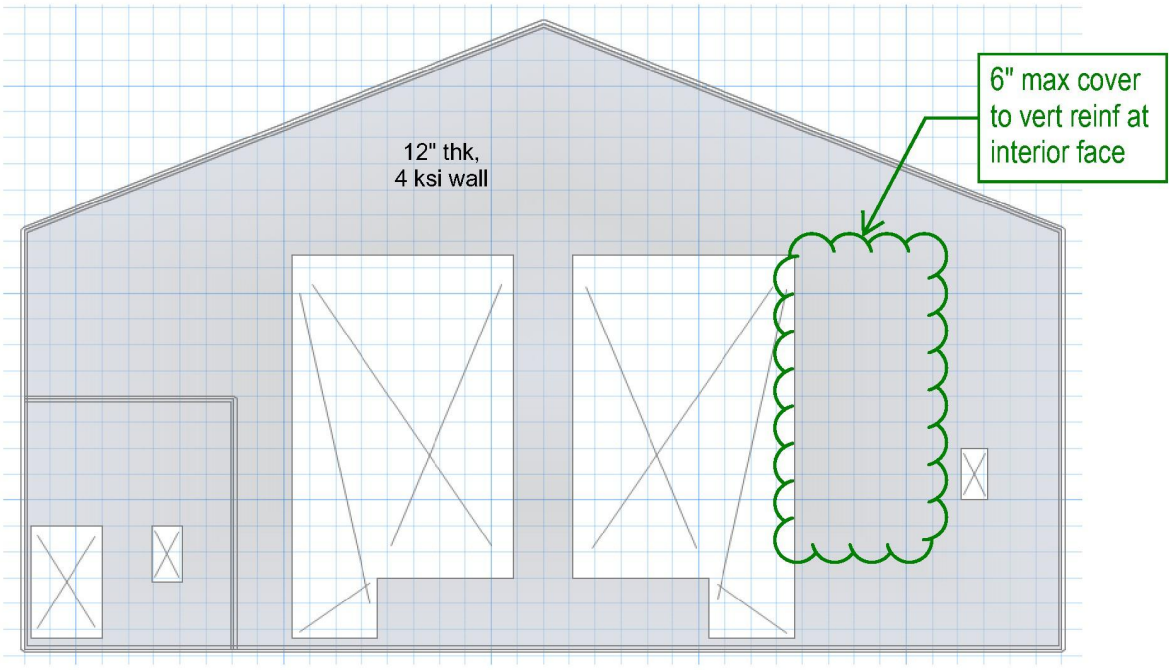
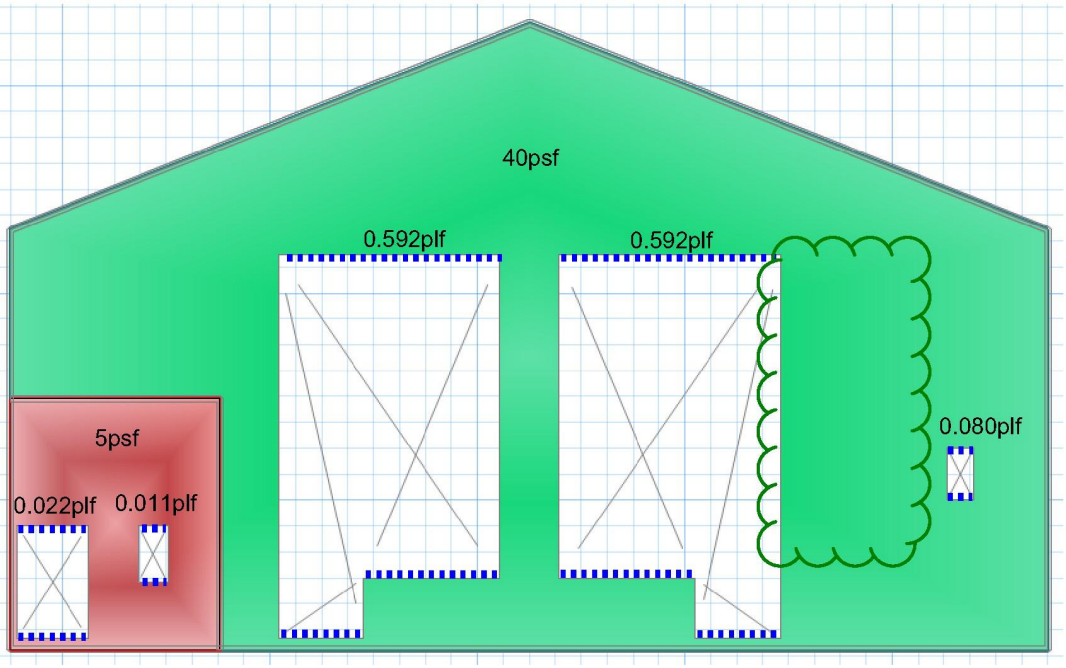


GEOMETRY



LOADING DIAGRAM

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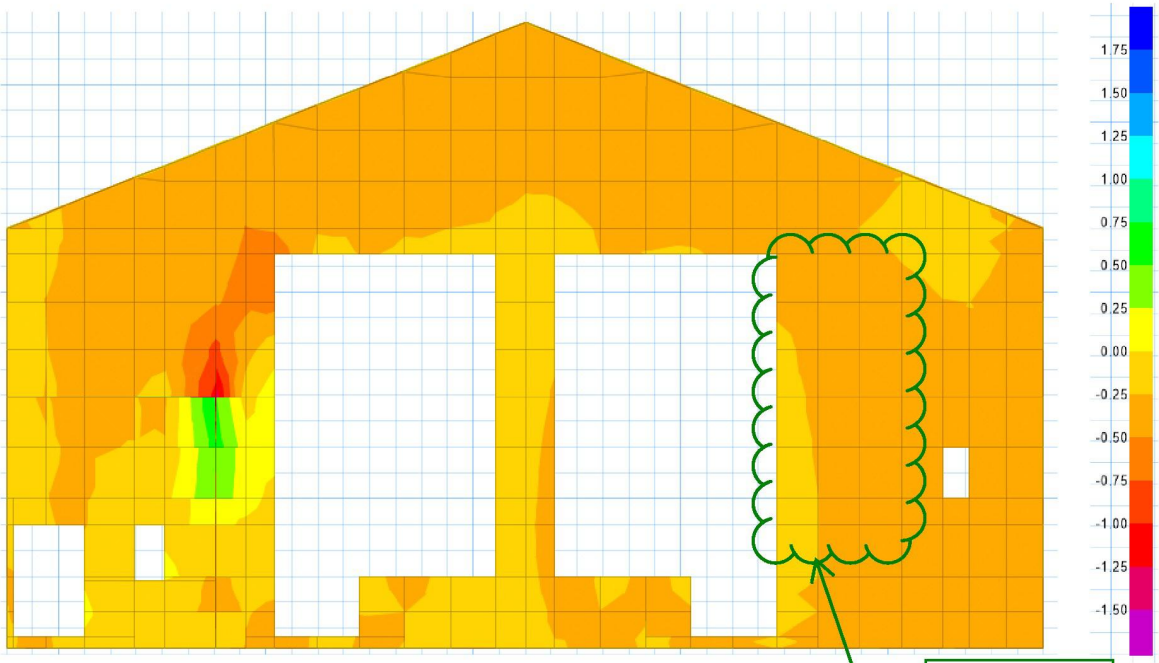
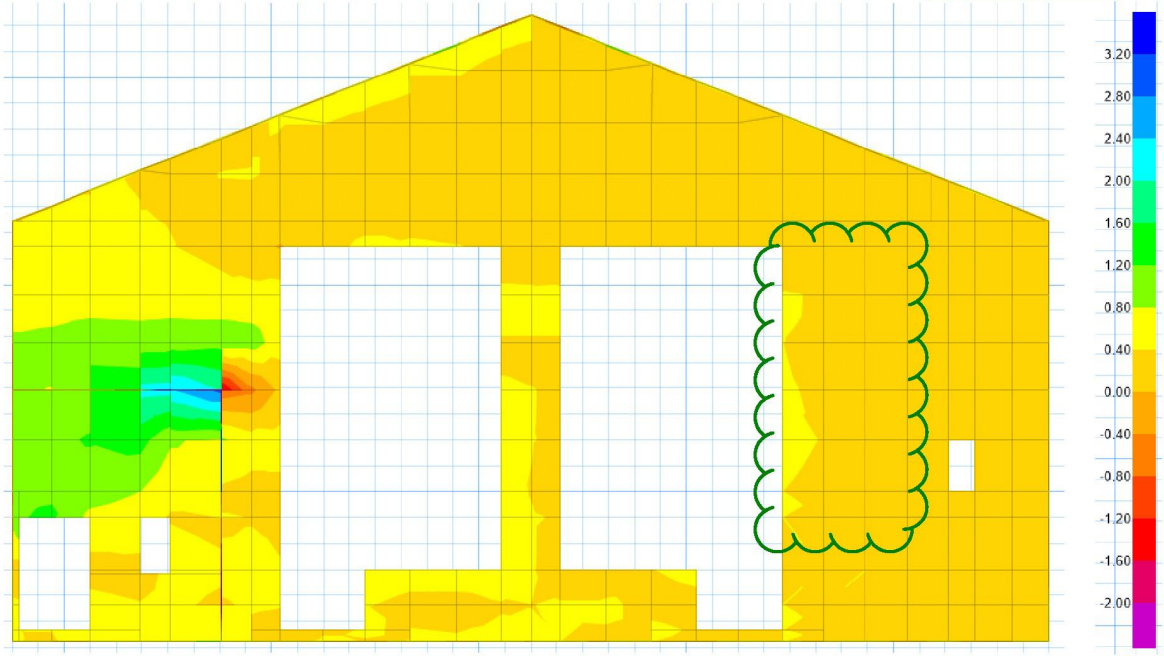
ICF Wall Summary - Calcs		CHECKED BY:	DRAWING #:



SHEAR - X (k/ft)



SHEAR - Y (k/ft)

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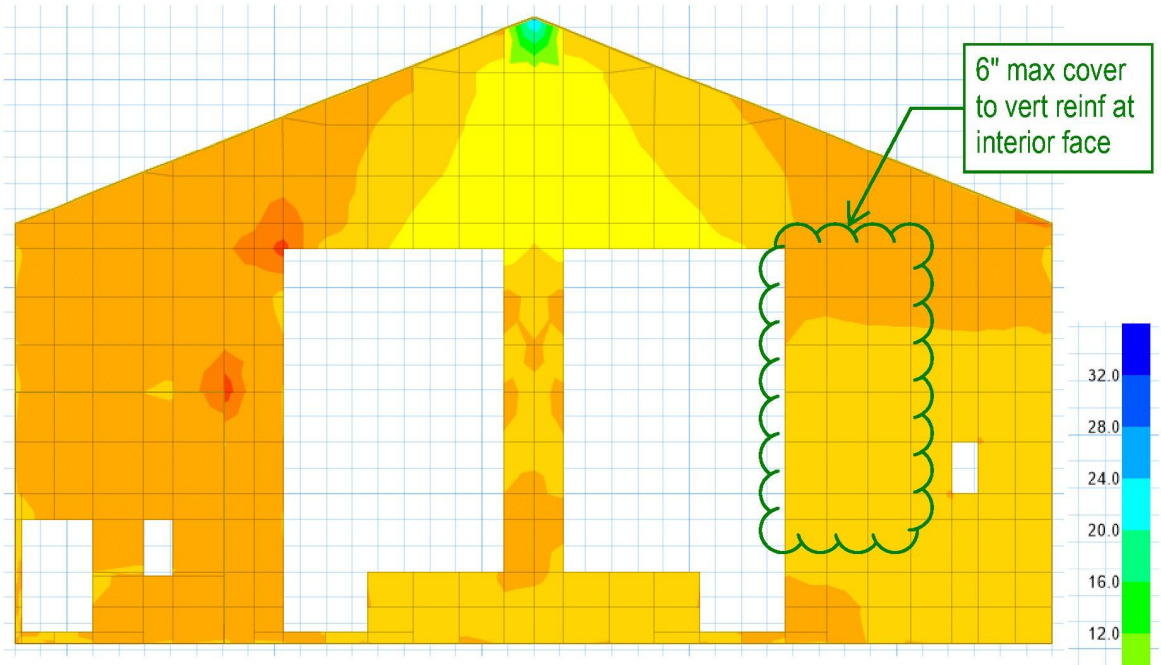
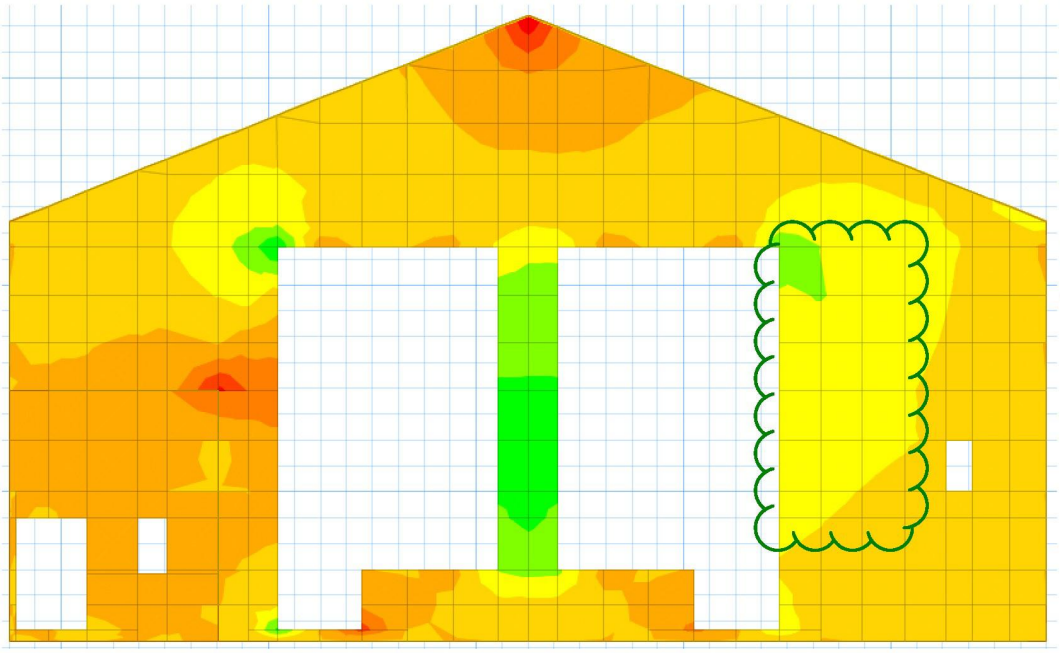
ICF Wall Summary - Calcs		CHECKED BY:	DRAWING #:



MOMENT - X (k-ft/ft)



MOMENT - Y (k-ft/ft)